ARTICLE 2

PROCEDURES FOR STRUCTURAL EVALUATION OF BUILDINGS

2.0 GENERAL

2.0.1 STRUCTURAL EVALUATION PROCEDURE

1. The structural evaluation process shall include the following steps:

    1. Site visit and data collection;

    2. Identification of building type;

    3. Completion of evaluation statements in appendix;

    4. Follow-up field work, if required;

    5. Follow-up analysis for "False" evaluation statements;

    6. Final evaluation for the building;

    7. Preparation of the evaluation report, and

    8. Submittal of evaluation report to OSHPD.

2. A general acute care hospital facility building may be exempted from a structural evaluation upon submittal of a written statement by the hospital owner to OSHPD certifying the following conditions:

    1. A conforming building as defined in Article 1, Section 1.01, may be placed into SPC 5 in accordance with Table 2.5.3 under of the following circumstances:
        a. The building was designed and constructed to the 1989 or later edition of Part 2, Title 24, and
        b. If any portion of the structure, except for the penthouse, is of steel moment resisting frame constuction (Building Type 3, or Building Types 4 or 6 with dual lateral system, as defined in Section 2.2.3) and the building permit was issued after October 25, 1994.
    2. All other conforming buildings as defined in Article 1, Section 1.01, may be placed into SPC 4 in accordance with Table 2.5.3, except those required by Section 4.2.10 to be placed in SPC 3 in accordance with Table 2.5.3, without the need for any structural evaluation.
    3. Nonconforming buildings as defined in Article 1, Section 1.01 may be placed into SPC 1 in accordance with Table 2.5.3 without any structural evaluation.

2.1 SITE VISIT, EVALUATION, AND DATA COLLECTION PROCEDURES

2.1.1 SITE VISIT AND EVALUATION

1. The evaluator shall visit the building to observe and record the type, nature, and physical condition of the structure.

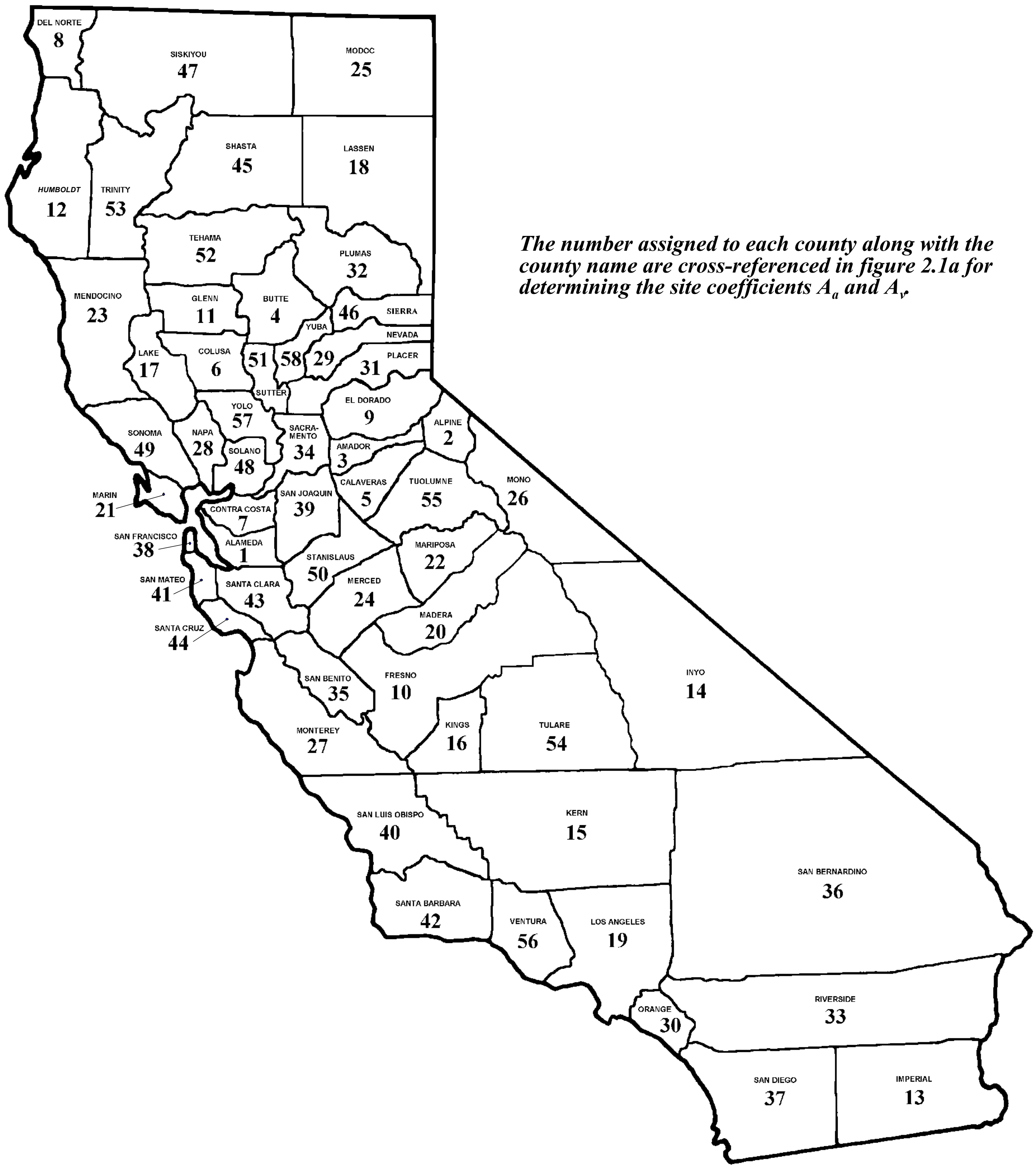
2. The evaluator shall review an *Engineering Geological Report* on site geologic and seismic conditions. The report shall be prepared in accordance with Title 24, Section 1634A.

Exceptions:

1. Reports are not required for one-story, wood-frame and light steel-frame buildings of Type II or Type V construction and 4,000 square feet or less in floor area;
2. A previous report for a specific site may be resubmitted, provided that a reevaluation is made and the report is found by the Office to be currently appropriate.

2. Establish the following *site and soil parameters*:

   a. The value of the effective peak acceleration coefficient (A_a) from Figure 2.1 and 2.1a;
   b. The value of the effective peak velocity-related acceleration coefficient (A_v) from Figure 2.1 and 2.1a;
   c. The soil profile type (S_1, S_2, S_3 or S_4) derived from the geotechnical report or from Table 2.1;
   d. The site coefficient, (S), from Table 2.1; and
   e. The ground motion parameters and near field effects in strong ground shaking required for the evaluation of welded steel moment frame structures per Sections 4.2.0.1, 4.2.0.2 and 4.2.10



The number assigned to each county along with the county name are cross-referenced in figure 2.1a for determining the site coefficients A_a and A_v.

FIGURE 2. 1 Index Map

FIGURE 2.1a
Effective peak acceleration coefficient (A_a) and effective peak velocity coefficient (A_v) for California

No.	County	EPA A_a	EPV A_v	No.	County	EPA A_a	EPV A_v
1	Alameda	0.40	0.40	30	Orange	0.40	0.40
2	Alpine	0.20	0.20	31	Placer	0.20	0.20
3	Amador	0.20	0.20	32	Plumas	0.20	0.20
4	Butte	0.20	0.20	33	Riverside	0.40	0.40
5	Calaveras	0.20	0.20	34	Sacramento	0.20	0.30
6	Colusa	0.20	0.30	35	San Benito	0.40	0.40
7	Contra Costa	0.40	0.40	36	San Bernardino	0.40	0.40
8	Del Norte	0.20	0.20	37	San Diego	0.40	0.40
9	El Dorado	0.20	0.20	38	San Francisco	0.40	0.40
10	Fresno	0.40	0.40	39	San Joaquin	0.30	0.30
11	Glenn	0.20	0.20	40	San Luis Obispo	0.40	0.40
12	Humboldt	0.20	0.30	41	San Mateo	0.40	0.40
13	Imperial	0.40	0.40	42	Santa Barbara	0.40	0.40
14	Inyo	0.40	0.40	43	Santa Clara	0.40	0.40
15	Kern	0.40	0.40	44	Santa Cruz	0.40	0.40
16	Kings	0.40	0.40	45	Shasta	0.20	0.20
17	Lake	0.30	0.30	46	Sierra	0.20	0.20
18	Lassen	0.20	0.20	47	Siskiyou	0.20	0.20
19	Los Angeles	0.40	0.40	48	Solano	0.40	0.40
20	Madera	0.20	0.30	49	Sonoma	0.40	0.40
21	Marin	0.40	0.40	50	Stanislaus	0.40	0.40
22	Mariposa	0.20	0.30	51	Sutter	0.20	0.20
23	Mendocino	0.40	0.40	52	Tehama	0.20	0.20
24	Merced	0.40	0.40	53	Trinity	0.20	0.30
25	Modoc	0.20	0.20	54	Tulare	0.40	0.40
26	Mono	0.40	0.40	55	Tuolumne	0.20	0.20
27	Monterey	0.40	0.40	56	Ventura	0.40	0.40
28	Napa	0.40	0.40	57	Yolo	0.20	0.30
29	Nevada	0.20	0.20	58	Yuba	0.20	0.20

TABLE 2.1
Soil Profile Types and Site Coefficients

Soil Profile Type	Profile with	Site Coefficient, S
S_1	Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second or by other appropriate means of classification. OR Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.0
S_2	Deep cohesionless or stiff clay conditions including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
S_3	Soft- to medium-stiff clays and sands characterized by 30 feet or more of soft- to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.	1.5
S_4	More than 70 feet of soft clays or silts characterized by a shear wave velocity less than 400 feet per second.	2.0

3. Assemble building design data including:

   a. Construction drawings, specifications, and calculations for the original building (Note: when reviewing and making use of existing analyses and structural member checks, the evaluator shall assess and report the basis of the earlier work);

   b. All drawings, specifications and calculations for remodeling work, and

   c. Material tests and inspection reports for nonconforming buildings. If the original drawings are available, but material test and inspection reports are not available, perform the testing program as specified in Section 2.1.2.2.

   If structural drawings are not available, the site visit and evaluation shall be performed as described in Section 2.1.1.4, and structural data shall be collected using the procedures in Sections 2.1.2.1 and 2.1.2.2.

4. During the site visit, the evaluator shall:

   a. Verify existing data;

   b. Develop other needed data (e.g., measure and sketch building as outlined in Section 2.1.2);

   c. Verify the vertical and lateral systems;

   d. Check the condition of the building, and

   e. Identify special conditions, anomalies, and oddities.

5. Review other data available such as assessments of building performance following past earthquakes.

6. Prepare a summary of the data using an OSHPD approved format.

7. Perform the evaluation using the procedures in Sections 2.2 through 2.5.

8. Prepare a report of the findings of the evaluation using an OSHPD approved format.

2.1.2 DATA COLLECTION

Building information pertinent to a structure's seismic performance, including condition, configuration, detailing, material strengths, and foundation type, shall be obtained in accordance with this section, and documented on drawings and/or sketches that shall be included with the structural calculations.

2.1.2.1 Building Characteristics

Characteristics of the building relevant to its seismic performance shall be obtained for use in the building evaluation. This shall include current information on the building's condition, configuration, material strengths, detailing, and foundation type. This data shall be obtained from:

1. Review of construction documents;

2. Destructive and nondestructive testing and examination of selected building components; and

3. Field observation of exposed conditions.

The characteristics of the building shall be established, including identification of the gravity- and lateral-load-carrying systems. The effective lateral-load carrying system may include structural and non-structural elements that will participate in providing lateral resistance, although these elements may not have intended to provide lateral resistance. The load path shall be identified, taking into account the effects of any modifications, alterations, or additions.

2.1.2.1.1 Nonconforming Buildings Without Construction Documents

Where the available construction documents do not provide sufficient detail to characterize the structure, the evaluation may be based on field surveys, summarized in as-built drawings. These drawings must depict building dimensions, component sizes, reinforcing information (for concrete and masonry elements), connection details, footing information, and the proximity of neighboring structures. All parts of the building that may contribute to the seismic resistance or that may be affected by the seismic response of the structure must be identified. The field survey shall establish the physical existence of the structural members, and identify critical load bearing members, transfer mechanisms, and connections. The survey shall include information on the structural elements and connector materials and details. Performing the field survey will entail removal of fireproofing or concrete encasement at critical locations to permit direct visual inspection and measurement of elements and connections. Non-destructive techniques such as radiographic, electromagnetic, and other methods may be used to supplement destructive techniques.

1. Steel Elements - Steel elements shall be classified by structural member type (e.g., rolled or build-up, material grade, and general properties). The survey shall note the presence of degradation or indications of plastic deformation, integrity of surface coatings, and signs of any past movement. For degraded elements, the lost material thickness and reduction of cross-sectional area and moment of inertia shall be determined. Visual inspection of welds shall be per American Welding Society D1.1, "Structural Welding Code-Steel". Structural bolts shall be verified to be in proper configuration and tightened as required in the AISC Steel Construction Manual. Rivets shall also be verified to be in proper configuration and in full contact, with "hammer sounding" conducted on random rivets to ensure they are functional. Nondestructive testing methods, such as dye penetrant and magnetic particle testing, acoustic emission, radiography, and ultrasound shall be used when visual inspection identifies degradation or when a particular element or connection is critical to seismic resistance and requires further verification. For buildings in which archaic cast and wrought irons are employed, additional investigations to confirm ductility and impact resistance shall be conducted.

2. Concrete Elements - The configuration and dimensions of primary and secondary structural elements shall be established. The configuration and condition of reinforcing steel shall be assessed, through removal of concrete cover and direct visual inspection, and through nondestructive inspection using electromagnetic, radiographic, and other methods. Critical parameters of the reinforcing system, such as lap splice length, presence of hooks, development within concrete, degree of corrosion, and integrity of the construction shall be established in sufficient detail to perform the structural evaluation.

3. Masonry Elements - The configuration and dimensions of masonry elements shall be established. The configuration and condition of reinforcing-steel shall be assessed, through removal of masonry cover and direct visual inspection, and through nondestructive inspection using electromagnetic, radiographic, and other methods. Critical parameters of the reinforcing system, such as lap splice length, presence of hooks, development within concrete, degree of corrosion, and integrity of the construction shall be established in sufficient detail to perform the structural evaluation.

4. Wood Elements - The configuration and dimensions of wood elements; the connections between wood elements; and the connections between wood and other structural components or elements such as concrete or masonry walls shall be established. The configuration and condition of wood members, including size, type, grade, condition and quality shall be assessed, though removal of finish materials, and examination of unfinished areas such as attics, crawl spaces, and basements. Critical connections and elements shall be visually inspected, using invasive procedures or removal of finishes where necessary. For shear walls, select locations shall be exposed to allow evaluation of sheathing material, nail size, spacing and installation (e.g., overdriven or nails that miss or split the framing members). The base connections of shear resisting elements shall be inspected and evaluated for their adequacy to connect the base of the structure to the foundation or structure below.

5. Foundation Elements - In the absence of dependable construction drawings, determination of the size and detailing of the foundation system requires invasive procedures. The evaluator shall select representative footings for exposure to establish footing size and depth. Conservative assumptions regarding the reinforcement may be made considering code requirements and local practice at the time of the design. In the absence of evidence to the contrary, it may be assumed that the foundation elements were adequately designed to resist actual gravity loads to which the building has been subjected.

2.1.2.2 Material Properties

The building evaluation shall be based on the strength and deformation properties of the existing materials and components. The strength of existing components shall be calculated using data on their configuration, obtained from the original construction documents, supplemented by field observations, and the test values of material properties. Where such effects may have a deleterious effect on component or structural behavior, allowances shall be made for the likely effects of strain hardening or degradation. Test values may be obtained from samples extracted from the structure, or from original materials and compliance certificates. The Office will determine the adequacy of the testing program.

2.1.2.2.1 Nonconforming Buildings With Construction Documents

The material properties for nonconforming buildings for which original construction documents of sufficient detail are available shall be confirmed by testing or from acceptable original materials and compliance certificates. If original materials and compliance certificates are available, they must provide the information specified in Items 1 through 4 of this section to be considered acceptable.

1. Steel Elements - The following properties are required for each member type (e.g., beams, columns, braces) and each steel grade used in the structure:

a) Ultimate tensile and yield capacities;
b) Modulus of elasticity, and
c) Deformation characteristics including mode of failure.

2. Concrete Elements - The following material properties are required for each member type (e.g., beams, columns, walls) in the structure:

a) Concrete compressive strength;
b) Concrete unit weight;
c) Concrete modulus of elasticity;
d) Reinforcing steel tensile yield point;
e) Reinforcing steel modulus of elasticity;
f) Reinforcing steel chemical composition and carbon equivalent, and
g) Reinforcing steel surface deformations.

3. Masonry Elements - The following material properties are required for each type of masonry in the structure:

a) Masonry compressive strength;
b) Masonry unit weight;
c) Masonry modulus of elasticity;
d) Reinforcing steel tensile yield point;
e) Reinforcing steel modulus of elasticity;
f) Reinforcing steel chemical composition and carbon equivalent, and
g) Reinforcing steel surface deformations.

4. Wood Elements - The following material properties are required for each type of wood element in the structure:

a) Identification of Wood Species, and
b) Grade Material. (Note: This may be established by visual inspection or stamped labels on the element)

2.1.2.2.2 Nonconforming Buildings Without Construction Documents

The material properties for nonconforming buildings for which original construction documents of sufficient detail are unavailable shall be confirmed by testing. The number and location of tests shall be selected so as to provide sufficient information to adequately define the existing condition of materials in the building. The evaluator shall determine the number and location of tests. The test locations shall be located throughout the entire building in those components which provide the primary path of lateral force resistance.

2.2 SELECTION AND USE OF EVALUATION STATEMENTS

2.2.1 IDENTIFICATION OF BUILDING TYPE

The evaluator shall determine the building type using the following procedure:

1. Identify the lateral-force-resisting system using text and drawings, including whatever components are available and effective to constitute a system. Prepare floor and roof plans, and elevations and sketches of the lateral force resisting system.

2. Select one or more of the 15 common building types which best characterize the structure(see Section 2.2.2 and 2.2.3 below). Structures with multiple lateral force resisting systems (different lateral systems in orthogonal directions, or structures where the system changes from level to level) may require the use of two or more building

types. In the case of hybrid structures or other buildings that cannot be adequately classified using the 15 building types, the alternative analysis procedure shall be used, or the building shall be placed in SPC“1".

3. Reproduce from the Appendix the list of evaluation statements. These statements shall be used for all types of buildings. Some statements on the list may not be appropriate. These statements may be marked "NA" as "not applicable". The Appendix also contains the set of evaluation statements that address foundations and geologic site hazards, and nonstructural elements.

2.2.2 USING THE GENERAL PROCEDURE

The general procedure involving use of the set of evaluation statements presented in the Appendix consists of the following steps:

1. Evaluate the basic building system according to the evaluation statements in Article 3;

2. Evaluate the vertical systems resisting lateral forces according to Article 4 (moment frames), Article 5 (shear walls), or Article 6 (braced frames) as appropriate. For buildings with a combination of vertical systems, each system in the building must be evaluated;

3. Evaluate the diaphragm or horizontal bracing system according to Article 7;

4. Evaluate the structural connections according to Article 8;

5. Evaluate the foundation and possible geologic site hazards according to Article 9;

6. Evaluate the nonstructural elements that involve immediate life-safety issues according to Article 10; and

7. Evaluate the critical nonstructural components and systems according to Article 11.

If a statement is found to be true, the condition being evaluated is acceptable according to the criteria of these regulations and the issue may be set aside. If a statement is found to be false, a condition exists that needs to be addressed further, using the specified analysis procedures. Analysis procedures are given Section 2.4. Each statement includes a reference to a particular section in Articles 3 through 10 where additional procedures for the resolution of the issues are given. The evaluator shall assemble the list of deficiencies and the results of the analysis and proceed to the final evaluation in Section 2.5.

2.2.3 COMMON BUILDING TYPES

The evaluator shall determine the type(s) of building being evaluated, choosing from among the following 15 common types:

1. **Building Type 1--Wood, Light Frame:** These buildings are typically small structures of one or more stories. The essential structural character of this type is repetitive framing by wood joists on wood studs. Loads are light and spans are small. These buildings may have relatively heavy chimneys and may be partially or fully covered with veneer. Lateral loads are transferred by diaphragms to shear walls. The diaphragms are roof panels and floors. Shear walls are exterior walls sheathed with plank siding, stucco, plywood, gypsum board, particle board, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.

2. **Building Type 2--Wood, Commercial and Industrial:** These are buildings with a floor area of 5,000 square feet or more and with few, if any, interior bearing walls. The essential structural character is framing by beams on columns. The beams may be glulam beams, steel beams, or trusses. Lateral forces usually are resisted by wood diaphragms and exterior walls sheathed with plywood, stucco, plaster, or other paneling. The walls may have rod bracing. Large exterior wall openings often require post-and-beam framing. Lateral force resistance on those lines may be achieved with steel rigid frames or diagonal bracing.

3. **Building Type 3--Steel Moment Frame:** These buildings have a frame of steel columns and beams. Lateral forces are resisted by the development of flexural forces in the beams and columns. In some cases, the beam-column connections have very small moment resisting capacity but, in other cases, the connections of some of the beams and columns were designed to fully develop the member capacities. Lateral loads are transferred by diaphragms to moment resisting frames. The diaphragms can be of almost any material. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction, and the frames consist of lines of strong columns and their intervening beams.

4. **Building Type 4--Steel Braced Frame:** These buildings are similar to Type 3 buildings except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.

5. **Building Type 5--Steel Light Frame:** These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

6. **Building Type 6--Steel Frame with Concrete Shear Walls:** The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Lateral loads are transferred by diaphragms of almost any material to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In "dual" systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidities. In this case, the walls would be evaluated under this building type and the frames would be evaluated under Type 3, Steel Moment Frames.

7. **Building Type 7--Steel Frame with Infill Shear Walls:** This is one of the older type of buildings. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels act as a diagonal compression strut between the intersections of the moment frame. If the walls do not fully engage the frame members (i.e., lie in the same plane), the diagonal compression struts will not develop. The peak strength of the diagonal strut is determined by the tensile stress capacity of the masonry panel. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The analysis shall be based on published research and shall treat the system as a composite of a frame and the infill. An analysis that attempts to treat the system as a frame and shear wall is not permitted.

8. **Building Type 8--Concrete Moment Frame:** These buildings are similar to Type 3 buildings except that the frames are of concrete. There is a large variety of frame systems. Older buildings may have frame beams that have broad shallow cross sections or are simply the column strips of flat-slabs.

9. **Building Type 9--Concrete Shear Walls:** The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low but reinforcing is light. Remodeling that entailed adding or

enlarging the openings for windows and doors may critically alter the strength of the modified walls. In newer buildings, the shear walls often are limited in extent, generating the need for boundary members and additional design consideration of overturning forces.

10. **Building Type 10--Concrete Frame with Infill Shear Walls:** These buildings are similar to Type 7 buildings except that the frame is of reinforced concrete. The analysis of this building is similar to that recommended for Type 7 except that the shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. Research that is specific to confinement of the infill by reinforced concrete frames shall be used for the analysis.

11. **Building Type 11--Precast/Tilt-Up Concrete Walls with Lightweight Flexible Diaphragm:** These buildings have a wood or metal deck roof diaphragm that distributes lateral forces to precast concrete shear walls. The walls are thin but relatively heavy while the roofs are relatively light. Tilt-up buildings often have more than one story. Walls can have numerous openings for doors and windows of such size that the wall behaves more like a frame than a shear wall.

12. **Building Type 12--Precast Concrete Frames with Concrete Shear Walls:** These buildings contain floor and roof diaphragms typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. The diaphragms are supported by precast concrete girders and columns. The girders often bear on column corbels. Closure strips between precast floor elements and beam-column joints usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. Lateral loads are resisted by precast or cast-in-place concrete shear walls.

13. **Building Type 13--Reinforced Masonry Bearing Walls with Wood or Metal Deck Diaphragms:** These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck with or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns; steel beams are supported by steel columns.

14. **Building Type 14--Reinforced Masonry Bearing Walls with Precast Concrete Diaphragms:** These buildings have bearing walls similar to those of Type 13 buildings, but the roof and floors are composed of precast concrete elements such as planks or tee-beams and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements may have a cast-in-place topping.

15. **Building Type 15--Unreinforced Masonry (URM) Bearing Wall Buildings:** These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. **Unreinforced Masonry Bearing Wall Buildings (TYPE 15) shall be assigned to SPC 1.** No further analysis is required.

2.3 FOLLOW-UP FIELD WORK

The first assessment of the evaluation statements may indicate a need for more information about the building. The evaluator shall make additional site visits, performing the necessary surveys and tests to complete the evaluation.

2.4 ANALYSIS OF THE BUILDING

The general requirements for building analysis (including the determination of force level, horizontal distribution of lateral forces, accidental torsion, interstory drift, and overturning) are summarized in this section. For cases where dynamic analysis is required, the general requirements are given in Section 2.4.10.

2.4.1 SCOPE OF ANALYSIS

When an evaluation statement is false and requires further analysis, the evaluator shall provide appropriate analyses that will cover the statement requirements. For the analysis, the evaluator will:

1. Calculate the building weights;
2. Calculate the building period;
3. Calculate the lateral force on the building;
4. Distribute the lateral force over the height of the building;
5. Calculate the story shears and overturning moments;
6. Distribute the story shears to the vertical resisting elements in proportion to their relative stiffness;
7. Examine the individual elements as required by the evaluation statements:
    a. Load and reaction diagrams for diaphragms and for the vertical resisting elements;
    b. Shearing stresses and chord forces in the diaphragm;
    c. Vertical components (walls and frames) and find the story deflections, member forces and deflections; and
    d. Total forces or deflections according to the specified load combinations.

For moment frames consisting of beams and columns, the distribution of story shears to the vertical lateral-force-resisting elements in that story may be in proportion to their relative stiffness. In multistory frame-shear wall structures or in structures where the vertical resisting elements have significantly different lateral stiffnesses, or where the stiffnesses of the vertical resisting elements change significantly over the height of the structure, an analysis of the entire structure under the prescribed lateral loads shall be performed.

2.4.2 DEMAND

All building components evaluated shall resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live, and snow loads. The following load combinations shall be used:

$$Q = 1.1Q_D + Q_L + Q_S \pm Q_E \tag{2-1}$$

or

$$Q = 0.9Q_D \pm Q_E \tag{2-2}$$

where

Q = the effect of the combined loads;

Q_D = the effect of dead load;

Q_L = the effective live load is equal to 25 percent of the unreduced design live load but not less than the actual live load;

Q_S = the effective snow load is equal to either 70 percent of the full design snow load or, where conditions warrant and approved by OSHPD, not less than 20 percent of the full design snow load except that, where the design snow load is less than 30 pounds per square foot, no part of the load need be included in seismic loading; and

Q_E = the effect of seismic forces.

The seismic portion of the demand (Q_E), is obtained from analysis of the building using the seismic base shear, (V), from Eq. 2-3.

2.4.3 SEISMIC ANALYSIS OF THE BUILDING

2.4.3.1 Base Shear

The seismic base shear determined from Eq. 2-3 is the basic seismic demand on the building. Element forces and deflections obtained from analysis based on this demand are the element demands, (Q_E), to be used in the load combinations of Eq. 2-1 and 2-2. The demands are modified in some cases as discussed in Section 2.4.11.

The seismic base shear, (V), in a given direction, shall be determined as follows:

$$V = C_s W \tag{2-3}$$

where:

C_s = the seismic design coefficient determined by Eq. 2-4 or 2-5;

W = the total dead load and applicable portions of the following:

- In storage and warehouse occupancies, a minimum of 25 percent of the floor live;
- Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater;
- Total operating weight of all permanent equipment; and
- The effective snow load as defined above in Section 2.4.2.

The seismic coefficient, (C_s), for existing buildings shall be determined as follows:

$$C_s = 0.67 \left(\frac{1.2\, A_v S}{R\, T^{2/3}} \right) = \frac{0.80\, A_v S}{R\, T^{2/3}} \qquad \text{(2-4)}$$

where

A_v = the peak velocity-related acceleration coefficient given in Figure 2.1 and 2.1a,

S = the site coefficient given in Table 2.1. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless OSHPD determines that soil profile S_4 may be present at the site, or in the event the soil profile S_4 is established by the geotechnical engineer.

R = a response modification coefficient from Table 2.4.3.1, and

T = the fundamental period of the building.

The value of C_s need not be greater than:

$$C_s = 0.85 \left(\frac{2.5\ A_a}{R} \right) = \frac{2.12\ A_a}{R} \qquad \text{(2-5)}$$

where

A_a = the effective peak acceleration coefficient given in Figure 2.1 and 2.1a.

2.4.3.2 Period

For use in Eq. 2-4, the value of T shall be calculated using one of the following methods:

Method 1. The value of T may be taken to be equal to the approximate fundamental period of the building, (T_a), determined as follows:

a. For buildings in which the lateral-force-resisting system consists of moment resisting frames capable of resisting 100 percent of the required lateral force and such frames are not enclosed or adjoined by more rigid components tending to prevent the frames from deflecting when subjected to seismic forces:

$$T_a = C_T h_n^{3/4} \qquad \text{(2-6a)}$$

where

C_T = 0.035 for steel frames,

C_T = 0.030 for concrete frames, and

h_n = the height in feet above the base to the highest level of the building.

b. As an alternate for concrete and steel moment resisting frame buildings of 12 stories or fewer with a minimum story height of 10 feet, the equation $T_a = 0.10N$, where N = the number of stories, may be used in lieu of Eq. 2-6a.

c. For all other buildings,

$$T_a = \frac{0.05\ h_n}{\sqrt{L}} \tag{2-6b}$$

where L = the overall length (in feet) of the building at the base in the direction under consideration.

Method 2. The fundamental period T may be estimated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. This requirement may be satisfied by using the following equation:

$$T = 2\pi \sqrt{\frac{\Sigma\ (w_i d_i^2)}{g\ \Sigma\ (f_i d_i)}} \tag{2-7}$$

The values of f_i represent any lateral force, associated with weights w_i, distributed approximately in accordance with the principles of Eq. 2-8, 2-9, and 2-10 below or any other rational distribution. The elastic deflections, d_i, should be calculated using the applied lateral forces, (f_i). The period used for computation of C_s shall not exceed $C_a T_a$ where C_a is given in Table 2.4.3.2.

TABLE 2.4.3.1
Response Coefficients*

R	C_d	System
		Bearing Wall Systems
6.5	4	Light-framed walls with shear panels
4.5	4	Reinforced concrete shear walls
3.5	3	Reinforced masonry shear walls
4	3.5	Concentrically braced frames
1.25	1.25	Unreinforced masonry shear walls
		Building Frame Systems
8	4	Eccentrically braced frames, moment resisting connections at columns away from link
7	4	Eccentrically braced frames, non-moment resisting connections at columns away from link
7	4.5	Light-framed walls with shear panels
5	4.5	Concentrically braced frames
5.5	5	Reinforced concrete shear walls
4.5	4	Reinforced masonry shear walls
3.5	3	Tension-only braced frames
1.5	1.5	Unreinforced masonry shear walls
		Moment Resisting Frame System
8	5.5	Special moment frames of steel
8	5.5	Special moment frames of reinforced concrete
4	3.5	Intermediate moment frames of reinforced concrete
4.5	4	Ordinary moment frames of steel
2	2	Ordinary moment frames of reinforced concrete
		Dual System with a Special Moment Frame Capable of Resisting at Least 25% of Prescribed Seismic Forces
		Complementary seismic resisting elements
8	4	Eccentrically braced frames, moment resisting connections at columns away from link
7	4	Eccentrically braced frames, non-moment resisting connections at columns away from link
6	5	Concentrically braced frames
8	6.5	Reinforced concrete shear walls
6.5	5.5	Reinforced masonry shear walls
8	5	Wood sheathed shear panels
		Dual System with an Intermediate Moment Frame of Reinforced Concrete or an Ordinary Moment Frame of Steel Capable of Resisting at Least 25% of Prescribed Seismic Forces
		Complementary seismic resisting elements
5	4.5	Concentrically braced frames
6	5	Reinforced concrete shear walls
5	4.5	Reinforced masonry shear walls
7	4.5	Wood sheathed shear panels
		Inverted Pendulum Structures
2.5	2.5	Special moment frames of structural steel
2.5	2.5	Special moment frames of reinforced concrete
1.25	1.25	Ordinary moment frames of structural steel

[1] Some building systems such as precast moment resisting frames are not listed in Table 2.4.3.1. When an unlisted building system must be evaluated, the evaluator shall perform an alternate analysis per Section 2.7 or place the building in SPC 1.

TABLE 2.4.3.2
Coefficient for Upper Limit on Calculated Period

A_v	C_a
0.4	1.2
0.3	1.3
0.2	1.4

2.4.3.3 Direction of Seismic Forces

Assume that seismic forces will come from any horizontal direction. The forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure except as discussed in Section 2.4.3.5.

2.4.3.4 Uplift

The beneficial effects of uplift at the foundation soil level may be considered, using the alternative analysis procedure.

2.4.3.5 Orthogonal Effects

The critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength should be used.

> *Exceptions:* Diaphragms and components of the seismic resisting system utilized in only one of the two orthogonal directions need not be designed for the combined effects.

2.4.3.6 Combinations of Structural Systems

When combinations of structural systems are incorporated into the same structure, the following requirements shall be satisfied:

1. **Vertical Combinations**.

    1. Structures not having the same structural system throughout their height shall be evaluated using the dynamic lateral force procedure.
       Exceptions:

       (1) Structures five stories or less without stiffness and strength irregularities may be evaluated using the equivalent lateral force procedures; and

       (2) Structures conforming to Section 2.4.3.6.2, below.

    2. A two-stage analysis may be used if a structure contains a relatively rigid base supporting a flexible upper portion and both portions considered separately can be classified as regular structures. The rigid base shall have a calculated natural period in each direction of not more than 0.06 seconds. The periods shall be evaluated using Eq. 2-7, or its equivalent, considering the total mass of the flexible upper portion concentrated at the top of the rigid base. The flexible upper portion shall be evaluated as a separate structure supported laterally by the rigid base. The rigid base shall be evaluated as a separate structure. The reactions of the flexible upper portion shall be applied at the top of the rigid base, amplified by the ratio of the R and C_d factors

of the superstructure divided by those for the base structure. The values of R and C_d for the base structure shall be greater than or equal to those used for the superstructure. The total lateral force on the base shall include the forces determined for the base itself.

2. **Combinations Along Different Axes**. If a building has a bearing wall system in only one direction, the value of R used for systems in the other direction shall not be greater than that used for the bearing wall system.

2.4.3.7 Vertical Distribution of Forces

The lateral force, (F_x), induced at any level shall be determined as follows:

$$F_x = C_{vx} V \tag{2-8}$$

and

$$C_{vx} = \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}} \tag{2-9}$$

where:

C_{vx}	=	vertical distribution factor,
V	=	total design lateral force or shear at the base of the building,
w_i and w_x	=	the portion of the total gravity load of the building (W) located or assigned to Level I or x,
h_i and h_x	=	the height (feet) from the base to Level I or x, and
k	=	an exponent related to the building period as follows:
		For buildings having a period of 0.5 seconds or less, $k = 1$
		For buildings having a period of 2.5 seconds or more, $k = 2$
		For buildings having a period between 0.5 and 2.5 seconds, k may be taken as 2 or may be determined by linear interpolation between 1 and 2

2.4.3.8 Horizontal Distribution of Shear

The story shear, (V_x), shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm.

2.4.3.9 Horizontal Torsional Moments

The increased shears resulting from horizontal torsion where diaphragms have the capability to transmit that torsion shall be evaluated. The accidental torsional moment shall be determined assuming displacements of the centers of mass each way from their calculated locations. The minimum assumed displacement of the center of mass at each level shall be five percent of the dimension at that level measured perpendicular to the direction of the applied force. For each element, the most severe loading shall be considered.

2.4.3.10 Overturning

Every structure shall be capable of resisting the overturning effects caused by earthquake forces specified. At any level, the overturning moments to be resisted shall be estimated using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the overturning moment shall be distributed to the various resisting elements in the same proportion as distribution of the horizontal shears to those elements. The foundations of buildings (but not the connection of the building to the foundation), except inverted pendulum structures, shall be evaluated for the foundation overturning design moment (M_f) at the foundation-soil interface determined using the overturning moment at the base with an overturning moment reduction factor of 0.75.

2.4.3.11 P-Delta Effects

The resulting member forces and moments and the story drifts induced by *P*-delta effects shall be considered in the evaluation of overall structural frame stability. *P*-delta need not be considered if the drift satisfies the "Quick Check for Drift" given in Section 2.4.7.

2.4.3.12 Foundations

The foundation shall be capable of transmitting the base shear and the overturning forces defined in this article from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

2.4.3.12.1 Soil Capacities

The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier, or caisson and the soil shall be sufficient to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For the load combination including earthquake, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil. Allowable soil capacities multiplied by a factor of 2.0 may be used, except that values for sliding friction may not be increased.

2.4.3.12.2 Structural Materials

The strength of concrete foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall be determined from the provisions of ACI 318. Reductions to foundation component capacities shall be made where components do not meet the requirements of ACI 318.

2.4.4 DEFORMATION AND DRIFT

When deformations and drift limits need to be checked, such as for frames failing the "Quick Check of Drift" and slender seismic resisting systems of any type, compute the elastic deformations caused by the required forces and then multiply by the factor C_d to determine the total deformations. Interstory drifts shall not exceed $0.0133h_{sx}$, where h_{sx} is the story height below level x. For purposes of this drift analysis only, it is permissible to use the computed fundamental period (T) of the building without the upper bound limitation specified in Section 2.4.3.2 when determining drift level seismic design forces.

2.4.5 DEMAND ON DIAPHRAGMS

The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements as determined by the evaluator. Permissible deflection permits the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

Floor and roof diaphragms shall be designed to resist a minimum force equal to $0.5A_v$ times the weight of the diaphragm and other elements attached to the building plus the portion of the seismic shear force at that level, (V_x), required to be transferred to the components of the vertical seismic resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

Diaphragms shall provide for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm as prescribed in Section 3.6.4 of the 1994 *NEHRP Recommended Provisions*.

2.4.6 DEMAND ON PARTS AND PORTIONS OF THE BUILDING

Parts and portions of structures and permanent nonstructural components and equipment supported by a structure and their attachments, as identified in the building evaluation procedures, shall be evaluated to verify that they are capable of resisting the seismic forces specified below. All attachments or appendages, including anchorages and required bracing, shall be evaluated for seismic forces. Nonrigid equipment, the structural failure of which would cause a life-safety hazard, also shall be evaluated.

Each element or component evaluated shall be capable of resisting a total lateral seismic force, F_p, where:

$$F_p = 0.67(A_v C_c W_c) \tag{2-10}$$

where

A_v = the velocity-related acceleration coefficient given in Figures 2.1 and 2.1a,

C_c = a coefficient given in Table 2.4.6, and

W_c = the weight of the element or component.

TABLE 2.4.6
Seismic Coefficient, C_c

		C_c
Parts of structure	Walls:	
	Unbraced (cantilevered parapets and walls)	2.4
	Other exterior walls at and above the ground floor	0.9
	All interior bearing and nonbearing walls and partitions	0.9
	Masonry or concrete fences over 6 feet high	0.9
	Penthouse (except where framed by an extension of the building frame)	0.9
	Connections for prefabricated structural elements other than walls with force applied at the center of gravity	0.9
Nonstructural components	Exterior and interior ornamentations and appendages	2.4
	Chimneys, stacks, trussed towers, and tanks:	
	Supported on or projecting as an unbraced cantilever above the roof more than one-half its total height	2.4
	All others including those supported below the roof with unbraced projection above the roof less than one-half its height or braced or guyed to the structural frame at or above its center of mass	0.9
	Mechanical, plumbing, and electrical equipment	0.9
	Anchorage for suspended ceilings and light fixtures	0.9

The NPC of the building shall be determined using the procedures in Article 11.

2.4.7 QUICK CHECKS OF STRENGTH AND STIFFNESS

Evaluation statements may require quick check estimates of the strength and stiffness of the building.

To check the average shear stress or drift for upper stories in addition to the first story, the story shear for an upper story may be approximated as follows:

$$V_j = \left(\frac{n + j}{n + 1} \right) \left(\frac{W_j}{W} \right) 1.2\,V \qquad (2\text{-}11)$$

where

V_j = maximum story shear at story Level *j*,

n = total number of stories above ground level,

j = number of story level under consideration,

W_j = total seismic dead load of all stories above Level *j* (see Section 2.4.1),

W = total seismic dead load, and

V = base shear from Eq. 2-3.

2.4.7.1 Story Drift for Moment Frames

The following equation for the drift ratio is applicable only to regular, multistory, multibay frames with columns continuous top and bottom:

$$DR = \left(\frac{k_b + k_c}{k_b \cdot k_c} \right) \left(\frac{h}{12\ E} \right) V_c C_d \qquad (2\text{-}12)$$

where

DR = drift ratio = interstory displacement divided by interstory height,

k_b = I/L for the beam,

k_c = I/h for the column,

h = story height (in.),

I = moment of inertia (in.^{4}),

L = center to center length (in.),

E = modulus of elasticity (ksi),

V_c = shear in the column (kips), and

C_d = deflection amplification factor from Table 2.4.3.1.

For reinforced concrete frames, use appropriate cracked section properties pursuant to ACI 318-95 or later. For other configurations of frames, compute the drift ratio from the principles of structural mechanics.

2.4.7.2 Shearing Stress in Concrete Frame Columns

The equation for a quick estimate of the average shearing stress, (v_{avg}), in the columns of concrete frames is as follows:

$$v_{avg} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \tag{2-13}$$

where

n_c = total number of columns,

n_f = total number of frames in the direction of loading,

A_c = summation of the cross sectional area of all columns in the story under consideration, and

V_j = story shear from Eq. 2-11.

Eq. 2-13 assumes that nearly all of the columns in the frame have similar stiffness. For other configurations of frames, compute the shear stress in the concrete columns from the principles of structural mechanics.

2.4.7.3 Shearing Stress in Shear Walls

The equation for a quick estimate of the average wall shear stress, (v_{avg}), is as follows:

$$v_{avg} = \frac{V_j}{A_w} \tag{2-14}$$

where

V_j = story shear at the level under consideration determined from Eq. 2-11 and

A_w = summation of the horizontal cross sectional area of all shear walls in the direction of loading. The wall area shall be reduced by the area of any openings. For masonry walls use the net area. For wood framed walls, use the length rather than the area.

The allowable stresses for the various types of shear wall building are given in: Section 5.1 for concrete shear walls, Section 5.3 for reinforced masonry shear walls, Section 5.4 for unreinforced masonry shear walls, and Section 5.6 for wood shear walls.

2.4.7.4 Diagonal Bracing

The equation for a quick estimate of the average axial stress in the diagonal bracing, (f_{br}), is as follows:

$$f_{br} = \left(\frac{V_j}{sN_{br}} \right) \left(\frac{L_{br}}{A_{br}} \right) \tag{2-15}$$

where

L_{br} = average length of the braces (ft),

N_{br} = number of braces in tension and compression if the braces are designed for compression; if not, use the number of braces in tension, if the braces are not designed for compression,

s = average span length of braced spans (ft),

A_{br} = the average area of a diagonal brace (in.^{2}), and

V_j = maximum story shear at each level (kips).

2.4.8 PROCEDURE FOR EVALUATING UNREINFORCED MASONRY BEARING WALL BUILDINGS

Unreinforced masonry bearing wall buildings shall automatically be placed in SPC 1.

2.4.9 ELEMENT CAPACITIES

Calculate element capacities on the ultimate-strength basis of the 1994 *NEHRP Recommended Provisions*.

When calculating capacities of deteriorated or damaged elements, the evaluator shall make appropriate reductions in the material strength, the section properties, and any other aspects of the capacity affected by the deterioration.

2.4.9.1 Wood

The basic document is Chapter 9 of the 1994 *NEHRP Recommended Provisions,* as modified in Section 5.6 of these regulations.

2.4.9.2 Steel

The basic document is Chapter 6 of the 1994 *NEHRP Recommended Provisions*, as modified in Articles 4 and 6 of these regulations.

2.4.9.3 Concrete

The basic document is ACI 318-89. Because this document is on an ultimate-strength basis, the 1994 *NEHRP Recommended Provisions* specifies special load factors that include the factor of 1.0 for earthquake effects (see Eq. 2-1 and 2-2).

2.4.9.4 Masonry

The basic document is Chapter 12 of the 1994 *NEHRP Recommended Provisions*, as modified in Article 5 of these regulations.

2.4.10 DYNAMIC ANALYSIS

Unless otherwise noted, the procedures given in Articles 3 through 10 use the equivalent lateral force procedure. The use of a dynamic analysis procedure is required for the following:

1) Buildings 240 feet or more in height;

2) Buildings with vertical irregularities caused by significant mass or geometric irregularities;

3) Buildings where the distribution of the lateral forces departs from that assumed in the equivalent lateral force procedure; and

4) Where required by the evaluation statements in Articles 3 through 10.

Dynamic analysis procedures shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation as specified in this section and shall be performed using accepted principles of dynamics. Structures that are evaluated in accordance with this section shall comply with all other applicable requirements.

2.4.10.1 Ground Motion

The ground motion representation shall be an elastic response spectra developed for mean values for the specific site, in accordance with the procedures in Title 24, Section 1629A.2.

2.4.10.2 Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to calculate the significant features of its dynamic response. A three-dimensional model shall be used when the dynamic analysis involves a structure with an irregular plan configuration and rigid or semirigid diaphragms.

2.4.10.3 Analysis Procedure

2.4.10.3.1 Response Spectrum Analysis

An elastic dynamic analysis of a structure shall use the peak dynamic response of all modes having a significant contribution to total structural response. This requirement may be satisfied by demonstrating that for the modes considered, at least 90%

of the participating mass of the structure is included in the calculation of response in each principal horizontal direction. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve that corresponds to the modal periods. Maximum modal contributions shall be combined in a statistical manner using recognized combination methods to obtain an approximate total structural response.

2.4.10.3.2 Scaling of Results

When the base shear for a given direction is less than that required by the equivalent lateral force procedure, the base shear shall be increased to the value prescribed in that procedure. All corresponding response parameters, including deflections, member forces and moments, shall be increased proportionately.

When the base shear for a given direction is greater than that required by the equivalent lateral force procedure, the base shear may be decreased to the value prescribed in that procedure. All corresponding response parameters, including deflections, member forces, and moments, may be decreased proportionately.

2.4.10.3.3 Post-Yield Analyses

Post-yield analyses of a simplified model of the building may be made to estimate the nonlinear displacements of the structural system. If the analyses is made with a two-dimensional planar model, the additive torsional displacement shall be established through methods that are equivalent to those used for response spectra analyses.

The displacements or rotations of structural members estimated by the post-yield analysis shall be compared with relevant experimental data to determine the adequacy of the member or system.

2.4.10.4 Torsion

The analysis shall account for torsional effects, including accidental torsional effects, as prescribed in Section 2.4.3.9. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations or by equivalent static procedures such as provided in Section 2.4.3.9.

2.4.11 ACCEPTANCE CRITERIA

The elements to be analyzed are specified in the procedures given in Articles 3 through 10. The total demand, Q, is calculated by Eq. 2-1 or 2-2 as modified below. The capacity, C, is calculated according to the procedures of Section 2.4.9. The basic acceptance criterion is:

$$Q \leq C \tag{2-17}$$

Where elements or portions of a lateral force resisting system are expected to behave in a less ductile manner than the system as a whole, the term Q_E in Eq. 2-1 or 2-2 shall be modified or special calculations be made to account for the different failure modes of the various elements. Modification of Q_E, and special calculation procedures and when they shall be used, are described in Articles 3 through 8.

If all significant elements meet the basic acceptance criteria as specified herein, no further analysis is needed.

2.4.12 ASSESSMENT OF ELEMENT DEFICIENCIES

The result of the checks specified in Articles 3 through 10 will show whether or not the elements meet the requirements of the 1994 *NEHRP Recommended Provisions* as modified herein.

For those elements not meeting the specified acceptance criteria, the relative hazard or seriousness of the deficiencies shall be assessed. Deficiencies shall be ranked according to:

1) Degrees of "overstress" (both total and seismic);

2) Element importance in the load path; and

3) Building, ductile and element stability.

2.5 FINAL EVALUATION

2.5.1 REVIEW THE STATEMENTS AND RESPONSES

Upon completion of the analysis and field work, the evaluator shall review the evaluation statements and the responses to the statements to ensure that all of the concerns have been addressed.

2.5.2 ASSEMBLE AND REVIEW THE RESULTS OF THE PROCEDURES

Upon completion of the procedures given in Articles 3 through 10, the evaluator shall assemble and review the results.

2.5.2.1 Q versus C

The criterion $Q \leq C$ is an indication of whether an element meets the requirements of the 1994 *NEHRP Recommended Provisions* as modified for these regulations. However, because Q involves gravity effects, the ratio of Q to C for an element must be considered in light of the seismic demand versus capacity in order to fully determine the seriousness of the earthquake hazard.

2.5.2.2 D_E/C_E Ratios

The severity of the deficiencies shall be assessed by listing the D_E/C_E ratios in descending order. The element with the largest value is the weakest link in the building. If the element can fail without jeopardizing the building, then the SPC may be based upon the element with the next lower ratio, and so on. Failure of an element will not jeopardize the building provided an alternate load path (neglecting the failed element) exists, and the vertical and lateral stability of the structure, or portions of the structure, is not impaired. The presence of an element with a D_E/C_E greater than one, where failure of that element will jeopardize the stability of the building or element, requires that nonconforming buildings be placed in SPC 1. For conforming buildings see the appropriate evaluation statement.

2.5.2.3 Qualitative Issues

Some of the procedures identify specific deficiencies without any calculation. These deficiencies will automatically place buildings in SPC 1, 3, or 4.

2.5.3 FINAL EVALUATION

The final evaluation will place the building in the appropriate the SPC (Table 2.5.3), based on a review of the qualitative and quantitative results of the procedures and the list of deficiencies. In general, an unmitigated “false” answer to an evaluation statement will lower the SPC of the Building. A "false" evaluation statement may be considered mitigated if the building, element or component s justified using the procedure outlined in the evaluation statement, or the effects of the condition are incorporated in the overall evaluation, as described in Section 2.5.2.2.

TABLE 2.5.3
STRUCTURAL PERFORMANCE CATEGORIES (SPC)

SPC	Description
SPC 1	Buildings posing a significant risk of collapse and a danger to the public. These buildings must be brought up to the SPC 2 level by January 1, 2008 or be removed from acute care service.
SPC 2	Buildings in compliance with the pre-1973 California Building Standards Code or other applicable standards, but not in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act. These buildings do not significantly jeopardize life, but may not be repairable or functional following strong ground motion. These buildings must be brought into compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, its regulations, or its retrofit provisions by January 1, 2030 or be removed from acute care service.
SPC 3	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, utilizing steel moment resisting frames in regions of high seismicity as defined in Section 4.2.10 and constructed under a permit issued prior to October 25, 1994. These buildings may experience structural damage which does not significantly jeopardize life, but may not be repairable or functional following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used to January 1, 2030 and beyond.
SPC 4	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, but may experience structural damage which may inhibit ability to provide services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used to January 1, 2030 and beyond.
SPC 5	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, and reasonably capable of providing services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used without restriction to January 1, 2030 and beyond.

2.5.3.1 Conforming Buildings

Conforming buildings, other than those of welded steel moment frame construction (Building Type 3 and possibly Building Types 4 and 6, if a dual system is present), without any unmitigated “false” evaluation statements shall be placed in SPC 5. Other conforming buildings shall be placed in the lowest SPC directed by the evaluation statements.

2.5.3.2 Nonconforming Buildings

An unmitigated "False" answer to any evaluation statement shall result in nonconforming buildings being placed in SPC 1, unless directed otherwise by the procedures for that particular evaluation statement. All other nonconforming buildings shall be placed in SPC 2.

2.6 THE FINAL REPORT

The report shall include the following elements:

1. A description of the building, including photographs, and sketches of the lateral force resisting system using an OSHPD approved format;

2. The set of statements from Appendix , with a synopsis of the investigation and supporting calculations that were made;

3. A list of the deficiencies that must be remedied in order to change statement responses from false to true;

4. The SPC for the building, with comments on the relative importance of the deficiencies; and

5. The NPC for the building.

2.7 ALTERNATIVE ANALYSIS

The owner of a building may elect to perform an Alternative Analysis, to evaluate a structure in more detail than that provided by the evaluation procedures specified in these regulations. The methodology of an Alternative Analysis must be approved in advance by OSHPD, and shall meet the following criteria:

1. Data collection on the structure and site conditions shall be performed in accordance with the appropriate Sections of Article 2 of these regulations. Depending upon the type of analysis to be performed, additional data regarding the as built condition and material properties may be required;

2. The Alternative Analysis shall be based on a site specific ground motion as specified in Section 2.4.10.1;

3. The analysis of the structure shall determine the distribution of strength and deformation demands produced by the design ground shaking and other seismic hazards. The analysis shall address seismic demands and capacities to resist these demands for all elements in the structure that either:

   # Are essential to the lateral stability of the structure (primary elements); or

   # Are essential to the vertical load carrying integrity of the building.

4. The analysis procedure may consist of a linear or nonlinear analysis. The analytical methods and acceptance criteria shall be reviewed and approved, in advance, by OSHPD.